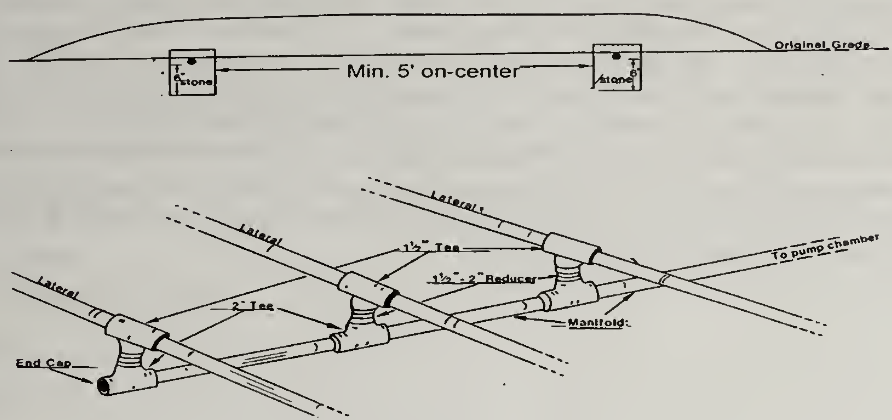
# FINAL REPORT

# DESIGN GUIDANCE FOR SHALLOW TRENCH LOW PRESSURE PIPE SYSTEMS




**Prepared for**

**Massachusetts Department of Environmental Protection**
**Grant Program 99-07/319**
**APRIL 2001**

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## PROJECT INTRODUCTION

Shallow-Trench Low-Pressure Pipe Wastewater Treatment Systems (ST-LPPs) are pressurized in-ground wastewater treatment and disposal systems utilizing narrow trenches. Effluent is dosed and uniformly distributed through small diameter pipes under low pressure. The systems are designed to utilize the upper soil horizons where biological treatment is the greatest and to provide the greatest separation between the system and groundwater, ledge, or other limiting layers. These alternative systems are significantly modified from conventional soil absorption systems to improve their treatment capability and allow them to function on difficult sites.

Shallow-Trench Low-Pressure Pipe Wastewater Treatment Systems are a shallow-placed subset of Low-Pressure Pipe Wastewater Treatment Systems (LPPs). Figure 1 shows a typical Low Pressure Pipe system layout. Specifically, as described herein and in comparison to conventional systems, these systems use full pipe flow pressure distribution and longer linear pipe runs. LPPs use low-pressure to dose effluent to the soil absorption field. Some low pressurized pipe systems, e.g. trickling filters, are pressurized but do not use dosing. Unlike a system that uses a pump to dose to a distribution box, LPPs are designed for uniform distribution of effluent. In uniform distribution, effluent is dosed under low pressure (typically 2.5 or more feet of distal head) through small diameter pipe (1.5 to 3 inches) using small orifice (0.25 to 0.375 inches) to ensure equal distribution of effluent throughout the soil absorption field.

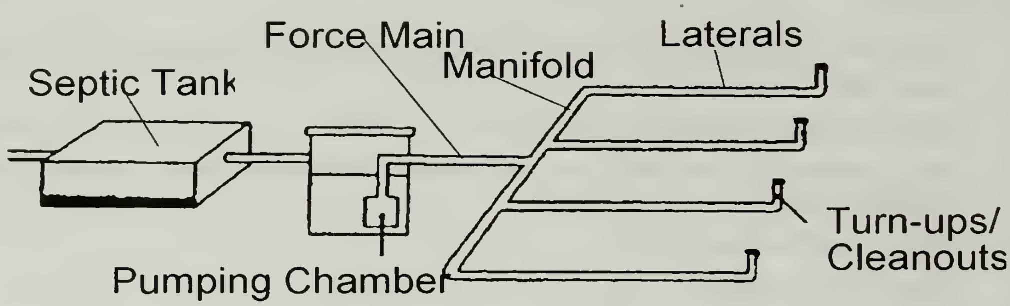


**Figure 1: Low-Pressure Pipe System (LPP) (US EPA, 1999)**

Uniform distribution in narrow shallow trenches minimizes short-circuiting of effluent to groundwater while maintaining a sustainable biological or clogging mat. When compared to conventional systems, ST-LPPs operate with a more effective separation to limiting layers. This type of system also maximizes the interaction of effluent with the soil matrix, thereby maximizing the physical filtering, biochemical transformations, and biological treatment potential.

This manual reflects best practices and research for the design of ST-LPPs. It is intended as guidance for professionals with significant experience in the design and installation of advanced soil absorption systems.

# LITERATURE REVIEW AND ANALYSIS

## Introduction

Low-Pressure Pipe Wastewater Treatment Systems, including shallow-trench systems, are very common in several regions of the United States. These systems have a relatively long track record as innovative systems. Between 1982 and 1988, over 5000 were installed in North Carolina alone (Hoover, 1990). Thousands more are now in use in North Carolina and elsewhere, especially throughout the south, midwest, and northwest.

The research literature identifies the potential ability of Shallow-Trench Low-Pressure Pipe Wastewater Treatment Systems to provide improved hydraulic performance and water quality treatment, especially when compared to conventional in-ground systems. Many of these benefits are provided by uniform distribution, and the resulting simplicity of construction. As documented and discussed on the following pages, ST-LPPs may provide the following benefits:

1. Uniform distribution minimizes problems from the clogging of biological mats. Uniform distribution minimizes short-circuiting of effluent through and along cracks and rocks in the soil profile to limiting soil layers. Non-invasive construction methods allow shallow placed trenches without the smearing of soil that can otherwise occur in shallow placed systems.

2. Uniform distribution allows the ability to spread effluent over a large area, reducing the risks of groundwater mounding and dilutes contaminant concentrations of effluent.[1]

3. Uniform distribution allows systems to wrap around trees and rocks, and placed at different elevations on sloping ground.

4. Shallow placement and uniform distribution allows effective use of the biologically, chemically, and physically active upper soil horizons. These soil layers provide the best level of treatment and are documented in research findings on LPPs, mounds, and at-grade pressurized systems.

5. Narrow trench design maximizes interface with native soil and maximize trench sidewall area. This allows systems to be built in native soil where other systems would clog, and, by using very narrow trenches, maximizes the sidewall contact with native soils.

The currently available literature suggests that LPP systems, including ST-LPP systems, are an improvement over conventional gravity systems in removal of pollutants and in hydraulic performance. Many of the benefits of ST-LPPs exist in any LPP design system, which use dosing and pressure

[1] LPPs share some of the benefits of spray and drip irrigation systems in that effluent is spread over a large area, providing dilution benefits. Unlike drip and spray irrigation, however, the risk of direct human or animal contact with sewage effluent is substantially decreased.

distribution to achieve uniform flows. However, some of the benefits require low-pressure distribution, shallow placement and narrow trenches, which are unique to ST-LPP systems.

## Hydraulic Performance

Numerous studies have demonstrated that pressure-distributed systems with uniform wastewater application function hydraulically in soils with moderate to severe limitations where conventional systems fail hydraulically. Hydraulic treatment is important as it relates to water quality treatment and protection from exposure to partially treated wastewater.

On-site sewage disposal systems can fail hydraulically for a number of reasons, including: siting systems in soils with high groundwater, densified soils, and excessive loading rates. One of the most common reasons for hydraulic failure is the over development of a biological mat, also known as a clogging mat or organic crust. Biological mats form in gravity-fed systems and some pressure-distributed systems. Biological mats are critical to treatment in gravity-distributed systems, but they are less critical in a uniform distribution system. A mature biological aids in slowing down effluent movement into the subsurface soils in conventional gravity systems and provides some distribution of effluent throughout the soil absorption field. Under high solids loading or where systems are placed in the groundwater, excessive biological mat build-up can result in hydraulic failure. The development of a biological mat for uniform distribution in a pressure distribution system is not necessary for uniform application of wastewater (Harkin et al., 1979).

The biological mat forms at the point where the sewage enters the native soil, typically wherever there is a film or textural change. Laak (1986) described the biological mat as a slime layer comprised of solids, mineral precipitates, microorganisms, and decomposition by-products. While the layer provides treatment and minimizes short-circuiting of effluent into the soil below the first few holes in a gravity-distributed soil absorption field, Otis (1985) reports its development can eventually cause hydraulic failure when its permeability drops to the point that effluent is no longer able to drain away from the soil absorption field.

In conventional gravity-fed systems and many pressure-dosed systems, distribution boxes (D-box) are typically used to equalize flow within a soil absorption field. In this case, effluent is drained or pressure dosed into a concrete or plastic distribution box. Outlet pipes are lined up at the same elevation to encourage equal flow into each drainage pipe. The reality, however, is that neither gravity-fed nor pressure-fed distribution boxes provide uniform distribution, and do not even equalize the flow into each of multiple drainage pipes (Gross et al., 1997).

Without uniform distribution, soil absorption fields can fail hydraulically. First, there is a tendency for portions of the soil absorption field to become overloaded. This causes the rapid development of an excessively thick biological mat. Finally, the system develops "creeping failure" or "progressive failure syndrome" as the biological mat spreads out to cover the entire soil absorption field. Once the biological

mat is formed, if effluent continues to be distributed by gravity, there is no significant opportunity for the system to rest and for natural biological processes to maintain or reduce the thickness of the mat.

Some designers have argued that systems can be designed to accommodate "creeping failure" by eliminating the distribution box and designing for serial distribution (Mellen, 1985). In serial distribution, lower trenches are fed from effluent running through the upper trench that is not absorbed in the upper trench. While this design removes the need for a distribution box, it does not reduce the build-up of a biological mat and may reduce permeability even further because the mat builds up even faster.

Hargett (1985) studied several LPP systems in poorly drained soils during a ten-month period, which included higher than average precipitation. In spite of severe continuous ponding and surcharging of the trenches, he reported no indication of severe clogging. Hargett (1985) suggests that the rapid recovery of the systems after ponding is due to shallow placement of the systems in the most biologically active layer, highest aeration levels and moisture fluctuations. Even on clay soils, with very high failure rates for conventional systems, LPPs have been demonstrated to perform hydraulically (Weymann et al., 1998). While these studies did not provide detailed water quality analysis, the hydraulic performance suggests that if hydraulic performance is acceptable, then water quality treatment will be possible. Weymann et al. (1998) reported increased hydraulic performance in LPPs compared to conventional in-ground systems in the same soils.

Simon and Reneau (1985) studied six prototype LPPs, each with slightly different designs. Unlike the LPPs generally described in the design literature, they studied systems with both gravity and pressure distribution, and systems with traditional use of aggregate and systems with a layer of sand between the aggregate and the soil. In examining non-traditional LPPs, the authors reported that uniform distribution prevented localized overloading, while systems with distribution boxes had localized overloading. In this study, systems were placed in clayey soils in an area with a history of conventional septic system failures. Although some ponding occurred, LPPs functioned hydraulically. No discernable benefit was obtained from using a layer of sand between the trench bottom and the aggregate in the soil absorption field.

Feiden et al. (1988) installed and monitored two residential shallow-trench LPPs in loamy and sandy till soils in clod windswept conditions in Vermont. Systems were operated and monitored for three years. They reported little or no biological mat build-up during the first three years of operation. No ponding or surfacing was reported in either system. While quantitative conclusions for water quality treatment were confounded, qualitative results suggested equal or better treatment as compared to a conventional subsurface soil absorption system located under the same site and operating conditions. Moreover, the cost difference was significantly lower for the ST-LPPs as compared to a sand mounded pressurized system.

Carlile (1986) reported that LPPs in large and small systems do not develop the slimes or clogging at the soil interface that is typical of gravity-fed in-ground systems. A biological mat or biological residue will form on most systems, however, well before there is actually clogging that significantly slows the long-term acceptance rate. Carlile (1986) did not compare LPPs to pressurized mound systems or other

pressurized systems. However, in comparison to conventional systems, he suggests that LPPs, sand filters and other pressurized systems develop smaller or no biological mat.

Hoover (1990) inspected 194 low-pressure pipe systems, as part of a follow-up inspection of 340 on-site systems in North Carolina. He noted a higher failure rate in LPPs than conventional systems due to failure of mechanical systems. Most problems, however, were not design or siting problems but were associated with poor maintenance, specifically failed switches, clogged orifices, and broken cleanouts and inspection ports. Hoover (1990) did not report increased incidence of hydraulic failure as compared to conventional subsurface soil absorption systems. This highlights that LPPs, as with most I/A systems, have higher maintenance requirements than conventional systems. Hoover's work further demonstrates the need for careful construction of all of the components in an LPP system, including such items as inspection ports, pressure pipe manufacture, and switch controls. Hover (1990) identified the second contributing factor to system failure as excessive loading rates in some LPPs. Hoover noted that the LPPs were installed only in those areas where conventional systems would not be permitted or were repairs to failed conventional subsurface soil absorption systems, and therefore were operated under more rigorous conditions.

Brown et al. (1991) reported high failure rates in first generation low-pressure pipe systems in Delaware. Brown suggested that failures were the result of poor installation and design problems. After design modifications were made, Brown (1991) reported lower failure rates in the second-generation of LPPs. Brown (1991) concluded that even lower failure rates are possible with properly designed and constructed ST-LPP systems.

Pressurized, or Wisconsin, mounds also generally have been reported to have excellent hydraulic performance. Within the mound itself, a mature organic crust or clogging layer rarely forms (Harkin et al., 1979). These systems typically do not develop a significant biological mat but do not have problems with short-circuiting or effluent distribution. Pressurized mounds have had reported hydraulic failures, however, typically due to improper fill material in the mound, improper siting and maintenance (Harkin et al., 1979).

Other than in extremely cold conditions not encountered in Massachusetts, freezing and related hydraulic problems are generally not a problem on properly designed systems, whether they are conventional systems (Harkin et al., 1979) or Low-Pressure Pipe systems (Feiden et al., 1988). Freezing may present some problems for mound systems because of their potential exposure, where winds can remove protective snow cover (Harkin et al., 1979). This is a minimal risk, however, even in very cold climates and is an almost non-existent risk in Massachusetts, which is more temperate than states like Wisconsin and Vermont that already have a very long history of mounded and pressurized mound systems.

## Treatment/Water Quality Performance

The literature is relatively limited with respect to water quality performance of LPP systems, as is true for reviews on conventional and proprietary alternative systems. The studies that have been performed,

however, indicate that LPPs compare favorably to conventional soil absorption systems. Cogger and Carlile (1984) reported that even under high water table conditions, when LPP trenches were saturated or just above groundwater, LPP systems reduced the amount of nitrate reaching the ground water and thereby provide better nitrogen treatment than conventional systems. They also reported lower median fecal coliform counts around LPP systems as compared with the continuously saturated conventional systems. Dosing provides greater periods of resting and therefore promotes aerobic conditions within the system, an important requirement for pathogen reduction (Cogger et al., 1982). Bomblat et al. (1994) also noted that LPPs appear to provide better nitrification and lower fecal coliform levels than conventional systems.

Pressurized distribution systems generally produce higher-quality effluent then the gravity distribution system in terms of Biochemical Oxygen Demand (BOD), total suspended solids (TSS) removal and nitrification (Harkin et al., 1979). Shallow-placed systems provide the greatest separation to limiting layers for a subsurface system while ensuring the maximum interaction of effluent and the most biologically active layer of native soil. All the findings on LPP systems and other uniform distribution system, including sand filters, which have been used in Massachusetts for some time, show a high degree of treatment, even though most of these systems have little or no biological mat. The following discussion provides a review of the removal of major wastewater constituents in ST-LPPs and LPPs.

## Nitrogen (N)

Nitrogen from wastewater is of concern both for human health and in coastal environments, for environmental damage. Contamination of drinking waters with nitrate ($NO_3^-$ ) increases the risks of methemoglobinemia and potential formation of carcinogenic nitrosamines. Nitrogen discharges to coastal regions is linked to eutrophication of waters and degradation of shellfish beds. Conventional soil septic tank soil absorption systems reduce total nitrogen between 0 and 35% (Eastburn and Ritter, 1985). Laak (1986) suggests that a well-designed mature conventional soil absorption system may be able to reduce nitrogen by up to 40-50% (Laak, 1986) but is not reported to do so in the same report. In conventional wastewater disposal systems, and even in many alternative systems, however, the primary reduction in nitrogen concentration is by dilution. This suggests that a mass based N reduction is very limited under conventional operating conditions. Advanced treatment systems have been developed to provide active N reduction processes including alternating cycles of aerobic and anaerobic conditions fostering nitrification and denitrification.

Dilution may lower N concentrations low enough to avoid health problems in most areas. Nitrogen reduction in domestic wastewater treatment may be problematic in nitrogen sensitive areas, such as coastal regions and near drinking water supplies. Depending on sensitive environmental receptors in a watershed, different simple nitrogen budgeting models can be used to determine when treatment of nitrogen becomes a critical system goal (Laak, 1986).

Domestic septic tank effluent averages 35-100 mg total Nitrogen per liter (US EPA, 1980) and is approximately 20% organically-bound nitrogen and 75% soluble ammonium-N ($NH_4$) (Harkin et al.,

1979). Denitrification and uptake are two processes that result in mass based reduction of N. While uptake is dependant on managing plants or biota that utilize N in biomass production, it is not a practical process for small scale wastewater treatment systems. Denitrification is the final process in the nitrogen cycle whereby nitrate ($NO_3^-$) is converted to $N_2$ gas under anaerobic conditions. Denitrification is the most effective method of removing nitrogen from wastewater effluent. Because such a small portion of septic tank effluent is nitrate ($NO_3^-$), denitrification cannot occur until after a nitrification phase of wastewater treatment process transforms reduced nitrogen to nitrate ($NO_3^-$).

Several innovative and alternative systems, and some conventional systems, utilize dosing of effluent, such as LPPs, pressurized sand mounds, sand filters, and other alternative systems. Dosing produces alternating aerobic and anaerobic conditions, which accelerates nitrification and denitrification and reduces the potential for nitrogen contamination (US EPA, 1999). While the biological mat that forms in gravity-fed systems will produce anaerobic conditions, the denitrification process cannot proceed due to the fact that nitrate-N is not present in significant quantities. Some researchers have reported that dosing increases the denitrification potential in conventional subsurface soil absorption systems (Eastburn and Ritter, 1985). Dosed systems may provide as much as provide 40 to 45% denitrification, even in systems that are not specifically optimized for nitrogen removal according to Eastburn and Ritter (1985). They also suggest that this rate of denitrification, however, may not be any more than that occurring in a conventional system with a mature biological or clogging mat.

Brunnell at al. (1999) reported no significant difference between nitrogen removal rates in eight pressurize-distributed and dosed systems designed for uniform flow (40% removal) and eleven conventional systems (48% removal). All of the systems examined were single-family homes on sandy soils. The uniform distribution systems in this study were not LPPs. However, this study demonstrates that pressure distribution, by itself, neither harms nor showed significant benefit for nitrogen removal.

Research available to document LPPs nitrogen removal rate is limited. Results on the performance of LPP systems are mixed and dependant on details of system use, design, soils and siting. LPPs are not specifically optimized for nitrogen removal and may not be more effective at nitrogen removal than alternative systems specifically designed to remove nitrogen.

Compared to conventional in-ground systems, LPPs do appear to provide some benefits for nitrogen removal. LPP systems provide excellent uniform distribution, aeration, and, as a result, nitrification, especially when compared to conventional systems (Bomblat et al., 1994 and Spooner et al., 1998). At least one study suggests that LPPs also provide significant denitrification, with denitrification occurring during seasonal high groundwater when there is inadequate vertical separation to groundwater (Stewart and Reneau, 1988). There is less evidence of this, however, in the collective literature, especially in systems with appropriate separation to groundwater.

Shaw and Turyk (1994) claimed no significant difference between the ability of pressurized mounds, in-ground pressurized systems, presumably including LPPs, and at-grade pressurized systems to remove nitrogen from effluent. Both pressurized mound systems and at-grade pressurized systems provide

nitrification and denitrification. Converse et al. (1994) suggests that dilution is a major factor for these systems discharging nitrogen to the groundwater at levels below the drinking water standard (10 mg/L-N or 40 mg/L $NO_3^-$-N). Dilution may be a more significant factor in ST-LPPs because they effectively spread wastewater out over a greater area, and thereby minimize nitrogen plumes, than non-pressurized systems and conventional soil absorption fields.

Many types of sand filters, fixed film reactors, and other systems are specifically designed to optimize nitrogen removal (US EPA, 1980). Harkin et al. (1979) reported that under field conditions an average of 44% of effluent nitrate was denitrified in the pressurized mound systems they studied. Unlike alternative systems designed to optimize denitrification, pressurized mounds do not focus on alternating aerobic and anaerobic cycles. Aerobic conditions in the mound, followed by a thin anaerobic cycle at the sand/sod/native topsoil interface, however, provide significant denitrification. Nitrification and/or denitrification may also occur in the native soil, depending on soil conditions.

Pressurized at-grade on-site systems are also not specifically designed to optimize nitrogen removal. These are systems utilizing uniform distribution in trenches placed immediately above a plowed soil layer. In essence these are pressurized-mound systems with no sand under the stone (Vermont, 1996 and Converse et al., 1991). Nitrification appears to be limited under the soil absorption field, and without nitrification, denitrification cannot occur (Converse et al., 1991).

Pressurized at-grade systems have much in common with pressurized-mounds and shallow-trench low-pressure pipe systems. All three systems use uniform distribution and take advantage of the most biologically active native soil layer. Pressurized at-grade systems, however, may often be lacking the aerobic conditions directly below the soil absorption field that is needed for nitrification to occur. When a pressurized at-grade system discharges just above the plowed native soil interface, there may be an anaerobic layer at the native soil interface which limits nitrification.

In contrast, a pressurized-mound system typically has a sequence of aerobic conditions in the sand fill, then a thin anaerobic layer at the plowed native soil interface, and then aerobic conditions in the upper couple of feet of native soil. A properly designed ST-LPP should have aerobic conditions from below the soil absorption field until a limiting layer is reached.

In any on-site wastewater treatment system, nitrification and denitrification is governed by moisture and available oxygen conditions and by the availability of biodegradable organic matter. The process is "highly sensitive" to oxygen levels, which is displaced as soil moisture rises (Converse et al., 1991). In a pressurized system, which has no significant biological mat to provide treatment, soil moisture and oxygen levels are especially important (Harkin et al., 1979).

Shallow trench-LPPs can avoid the biological mat and may provide greater nitrogen treatment potential than a conventional system. In addition, none of the studies reviewed suggest any possible detrimental effect of nitrogen treatment as opposed to using a conventional system. LPPs, however, should not be

assumed to provide the nitrogen reducing opportunities of a well-designed pressurized sand mound or other alternative systems specifically designed to optimize denitrification.

## Phosphorus (P)

The phosphorus concentration in domestic septic tank effluent ranges from 6 to 29 mg per liter (US EPA, 1980). Most phosphorus in septic tank effluent (85%) is in the orthophosphate form. The remaining phosphorus may be associated as organic or precipitated forms and is likely to be converted to orthophosphate over time (Harkin et al., 1979).

The LPP technologies can substantially reduce total phosphorous, through soil chemical adsorption, plant uptake, and dilution in ground water. One study, using chloride/pollutant ratio analysis, indicated that much of the total phosphorus reduction is due to system treatment and not simply dilution (Spooner et al., 1998).

A field examination of pressurized, or Wisconsin, mound systems reported higher phosphorus discharge levels than conventional systems, primarily because actual loading rates where higher then the systems were designed to handle (Harkin et al., 1979). In addition, many of the systems utilized large dose volumes, which reduced the efficiency of phosphorus removal mechanisms. Systems with smaller doses had lower phosphorus discharge levels. Throughout the study, however, phosphorus was generally not reported to be a problem.

## Pathogens and biological contaminants

Biological contaminants from wastewater, including bacteria, viruses, protozoa, and parasites, are significant health risks and environmental concerns. Fecal coliforms, an indicator of biological contaminants from warm-blooded animals, are in the range of $10^9$ to $10^{12}$ organisms/L (US EPA 1980). Cogger et al. (1983) reported that a minimum of 24 inches (60 cm) of unsaturated soil, even sandy soil, is enough to remove viruses and other constituents of effluent. Other researchers have indicated that 48 inches (120 cm) is needed to remove viruses. Transport of viruses will vary with different soils, loading rates, and system designs. While there has been some research to find reliable viral indicators for modeling viral transport, there are significant limitations to these indicators. Bechdol et al. (1994) reported that viruses are not as likely to be filtered as bacteria and can survive in the soil for relatively long time periods, making non-viral indicators less useful as a surrogate for viruses. Virus modeling and testing of viruses in laboratory work is an area requiring additional research and development.

Bomblat et al. (1994) reported fecal coliform levels significantly lower under an LPP then under a comparable conventional system. They report this to be the result of more uniform distribution of effluent and a greater depth to groundwater because the system can be placed higher in the soil profile. Shallow-trench LPPs maximize separation distance to groundwater by placing the system shallower than conventional subsurface soil absorption systems. ST-LPPs utilize the unsaturated soil more effectively than gravity systems through their use of uniform distribution. Shallow-trench low-pressure pipe systems are very effective at removing biological contaminants, even on sites where soil conditions do not allow

conventional systems. Ijzerman et al. (1992) reported that shallow trenches allow effluent to be treated in an aerobic and biologically active layer, which result in highly effective treatment.

Pressurized, or Wisconsin, mounds provide the same or greater treatment of fecal indicator organisms than conventional systems (Harkin et al., 1979). This is noted to be due to a combination of uniform LPP distribution and the use of unsaturated sand above native soil. Pressurized-mound systems are effective at treating fecal indicator organisms, though depending on the soil type, they may not always be better at removing these organisms than a conventional gravity system with a clogging mat (Converse et al., 1994). Although pressurized, or Wisconsin, mounds are not exactly the same as ST-LPPs, the treatment benefits of pressurized distribution, which is similar in both types of systems, have been studied more extensively in mounds than in ST-LPPs.

## Total Suspended Solids and Biochemical Oxygen Demand

Total suspended solids (TSS) in domestic wastewater are generally between 680 and 1000 mg/L (US EPA, 1980). A typical septic tank may be capable of reducing solids by as much as 60% with a two-day residency time. Solids in septic tank effluent are removed readily by filtration and biological degradation. Solids can cause environmental problems from effluent discharged, directly or indirectly, into surface waters. The primary problem with TSS in a sewage disposal system, which discharges into a soil absorption system, is system clogging and failed hydraulic performance. Biologically degradable solids or the organic carbon fraction in wastewater is readily removed by microbial activity under aerobic conditions. Under anaerobic condition microbial degradation of solids is markedly slower generally resulting in accumulation of solids in a constantly loaded system. LPPs and conventional systems are both highly effective at removing carbon under aerobic conditions (Bomblat et al., 1994). Uniform-distributed systems generally have far less soil clogging than gravity-distributed systems. As a result, suspended solids should not create a problem in appropriately designed ST-LPPs. In general, Harkin et al. (1979) suggest solid reduction in pressurized LPP systems is adequate.

Typical domestic wastewater effluent has $BOD_5$ concentrations between 200-290 mg/L and Chemical Oxygen Demand concentrations are 680-730 mg/L (US EPA, 1980). Septic tank effluent may have roughly half these concentrations when passed through a septic tank with a two-day residency time. Biochemical oxygen demand (BOD) is a major environmental problem for effluent discharged into surface waters resulting in depleted oxygen concentrations. Conventional subsurface soil absorption systems under aerobic conditions and proper loading rates will typically treat all in-coming BOD. However, under stressed conditions, such as excessive loading rates or groundwater inundation, BOD loadings will increase the thickness of the biological mat resulting in hydraulic failure. As discussed above, uniform distribution reduces this likelihood of clogging.

## Design Parameters

"The Design and Installation of Low Pressure Pipe Waste Treatment Systems" (Cogger et al., 1982) was the first widely cited design manual on LPPs. Cogger et al. (1982) gave design specifications calling for a minimum of 12 inches (30 cm) of unsaturated soil between the bottom of an LPP and seasonal high ground water or any restrictive horizon. They also reported that shallow and narrow trenches and uniform distribution allow relatively high loading rates. These loading rates can be presented in design standards and regulations in two ways, although both can effectively be used to mandate exactly the same design standard and it is easy to convert from one system to the other. Loading rates can be presented based on the basal area of the entire ST-LPP, including trenches and the area between the trenches. Trench separation was suggested to be no greater than five feet on-center for narrow LPPs (Cogger et al., 1982). Alternatively, loading rates can also be calculated based on a linear loading rate, with systems being required to maintain a minimum separation distance.

Linear loading rates are easier for designers to understand. The linear loading rate method makes it easier to design a system when there may be a greater separation between trenches. For example, flexibility is provided when curving trenches to avoid rocks or trees or to remain at the same elevation. In either method, system loading divided by the loading rate determines the length of the trenches. With one-foot wide trenches placed five feet on-center, linear loading rates will be five times the basal loading rate. Variations in trench width and separation between trenches will obviously change this ratio.

The literature and other state code consist of numerous design specifications. In this report, we developed a table for comparisons with the Massachusetts Department of Environmental Protection's Title 5 (MA DEP, 1995) soil class loading rates. Table 1 lists loading rates for ST-LPPs under different trench configurations based on the MA DEP regulations.

**Table 1.** Maximum loading rates based on MA DEP Title 5 (310 CMR §15.242)

| *All loading rates calculated from Massachusetts Title 5* | Title 5 (310 CMR 15.243) class and soil texture | | | |
|---|---|---|---|---|
| | CLASS I | CLASS II | CLASS III | CLASS IV* |
| | Sands, Loamy Sands | Sandy Loams, Loams | Silt Loams | Clays, Silty Clay Loams |
| Loading Rate in trench bottom area, no sidewall credit (GPD/ft$^2$) | 0.66 to 0.74 | 0.33 to 0.60 | 0.29 to 0.37 | 0.15 |
| Basal loading rate for entire system footprint, assuming 12" wide trenches five feet on-center and 6" sidewall invert (GPD/ft$^2$). | 0.26 to 0.30 | 0.13 to 0.24 | 0.12 to 0.15 | 0.06 |
| Linear loading rate in 9" wide trench with 6" sidewall invert, (GPD/ft). | 1.16 to 1.30 | 0.58 to 1.05 | 0.51 to 0.65 | .26 |
| Linear loading rate in 12" wide trench with 6" sidewall invert, (GPD/ft). | 1.32 to 1.48 | 0.66 to 1.20 | 0.58 to 0.74 | 0.30 |

*Massachusetts only allows the use of Class IV soils and soils with percolation rate slower than 30 minutes per inch for upgrades of existing systems or with a variance for new systems.

Table 2 provides a summary of basal and linear loading rates reported the literature and values proposed in the design guidance for Massachusetts I/A applications. The table also includes Title 5 loading rates for comparison. When Title 5 was written, unlike all of the other cited regulations and articles, however, LPPs were not considered in the creation of loading rates. Where researchers used only basal loading rates (including the area between the trenches), we converted the figures to linear loading rates to allow easy comparisons.

Most regulations and research articles use basal area or linear loading rates, and not square foot loading rates, because they do not provide higher loading rates for wider trenches. As a result, in Table 2 it is not possible to separate loading rates based on trench width (except for from Title 5 and from this paper).

**Table 2.** Maximum loading rates in gallons per day (GPD) based on the entire system basal area (including area between trenches) and on linear loading*

| REFERENCE LITERATURE | Title 5 (310 CMR 15.243) class and soil texture | | | |
|---|---|---|---|---|
| | CLASS I | CLASS II | CLASS III | CLASS IV |
| | Sands, Loamy sands | Sandy loams, Loams | Silt loams | Clays, Silty clay loams |
| Cogger et al., 1982 | Data not available | Data not available | Data not available | Basal: 0.10-0.20<br>Linear: 0.25-0.50 |
| Carlile, 1979 | Data not available | Data not available | Data not available | Basal: 0.10-0.20<br>Linear: 0.25-0.50 |
| Feiden et al., 1988 | Data not available | Basal: 0.40<br>Linear: 1.8 | Data not available | Basal: 0.10<br>Linear: 0.25 |
| Hargett (1985)-<br>*(interpretation)* | Data not available | Data not available | Data not available | Basal: 0.06<br>Linear: 0.30 |
| NC Health, 1999 | Basal: 0.4-0.6<br>Linear: 2.0-3.0 | Basal: 0.3-0.4<br>Linear: 1.5-2.0 | Basal: 0.15-0.3<br>Linear: 0.75-1.5 | Basal:0.05-0.20<br>Linear: 0.25-1.00 |
| Mass. DEP Title 5 for conventional systems (See Table 1 above.) | Basal: 0.25-0.30<br>Linear-9" trench: 1.16- 1.30<br>12" trench: 1.32-1.48 | Basal: 0.13-0.24<br>Linear-9"trench: 0.58-1.05<br>12" trench: 0.66-1.2 | Basal: 0.06-0.15<br>>30 M/I for upgrades<br>Linear-9" trench: 0.26-0.65<br>12" trench: 0.30-0.74 | Basal: 0.06<br>Linear-9" trench: 0.26<br>12" trench: 0.30<br>For upgrades only |
| LPP Design Guidance (from Appendix A) | Linear:<br>9" trench: 1.3<br>12" trench 1.4 | Linear:<br>9" trench: 1.1<br>12" trench: 1.2 | Linear:<br>9" trench: 0.85<br>12" trench: 0.75 | Linear:<br>9" trench: 0.25<br>12" trench: 0.3 |

*Findings are only loosely comparable both because of differences in required design flows calculations (e.g. 110 to 150 gallons per day per bedroom), which effectively change the loading rate, and differences in how percolation rate is determined.

As shown in Table 2, Massachusetts requires lower loading rates for conventional soil absorption systems than the cited LPP regulations and research articles, except in clays and silty clay loam soils. The loading rates provided in the LPP Design Guidance (Appendix A), and shown on the above table, are lower than that in the cited regulations and research articles. These Design Guidance loading rates are all within the range of Title 5.

Feiden et al. (1988) reported that two experimental ST-LPPs on silt loam and sandy till soils could function hydraulically in a cold Vermont climate, even at higher loading rates than recommended by Cogger et al. (1982). Shallow-Trench LPPs were used on these sites instead of pressurized-mounds.

Although clay soils present difficult hydraulic conditions for any subsurface soil absorption system, unsaturated fine textured soils are typically capable of providing a higher degree of wastewater renovation than other soils and will protect groundwater from contamination. Cogger et al. (1982) provides loading rates for LPP systems in clay soils, which create larger LPP systems than in other soils. It can be, however, difficult to design LPP systems in clay soils following all of Cogger's criteria. From a design standpoint, as system size increases, it is necessary to increase pump size, the number of manifolds, and diameter of the delivery pipe, and/or to decrease size of orifices and increase orifice spacing.

## State Regulations

North Carolina's regulations (NC Environmental Health, 1999), allow LPPs based only on the quality of the upper two feet of soil, regardless of site limitations below that level. Trenches must be a minimum of 8" wide and must have a minimum of 5" of stone below the pipe. While the bottom of the trench is typically 12"-15" below grade, it can be shallower. Trenches must be placed five feet on-center.

Delaware (Brown et al., 1991) also allows LPP systems, with a required 27" of separation to a limiting layer. To address problems resulting from the performance of first-generation LPPs, Delaware requires trenches a minimum of 8" wide for slowly permeable soils and 12" wide for faster soils. Trenches must be placed five feet on-center.

Pennsylvania (Pennsylvania Dept. of Environmental Conservation, 2000) allows shallow-placement pressure-dosed systems as an alternative system. These systems have much in common with ST-LPP, except that Pennsylvania allows system width of between one and six feet. Pennsylvania requires that these systems meet all of the separation requirements of conventional systems, but because the system can be shallower than 12" deep, it is easier to obtain the necessary separation.

Vermont's regulations (Vermont Dept. of Environmental Conservation, 1996) do not allow LPPs, but they do allow at-grade pressurized systems, which share many features with LPPs. These are conventional systems placed at grade such that the bottom of the stone is at or below grade, in topsoil that is simply tilled. Although not an ST-LPP, the ability of these systems to work is based on the ability of pressure distribution to ensure even distribution of effluent, and on the ability of the upper soil layers to provide the superior wastewater renovation capabilities. Vermont regulations require the equal distribution and pressure distribution components of LPPs and are used in a variety of alternative and conventional systems. Vermont's wide success with pressure distribution suggests the ability of pressure distribution to prevent short-circuiting and ensure the most effective use of based treatment systems.

Washington (1995) does not specifically discuss LPPs. Except in the case of single-family homes in semi-arid rural areas on large lots. Washington's regulations require the use of pressure-distribution

systems for any sewage disposal system with less 3 feet vertical separation to limiting layers. With the same exception, Washington also requires the use of pressure-distribution systems for any site on coarse sands and all extremely gravelly soils.

## Pressure System Details

Pressure distribution systems can fail due to clogging of orifices with the low-pressure distribution lines or laterals. While these problems can be significant and can lead to increased maintenance costs, these systems are generally robust and design and installation practices can be modified to reduce such problems (Glotfelty et al., 1997).

Researchers have noted some problems and design fixes for low-pressure distribution networks:

1. Orifices clog; requiring pipe cleanouts or turn-ups to allow cleaning of pipes. Given the number of orifices, however, this clogging may not always require maintenance (Glotfelty et al., 1997), but can significantly reduce flow rates (Hoover et al., 1991). Pipes should be designed for easy cleanouts and adequate cycle counters to determine when a system might need to be maintained (Glotfelty et al., 1997 and Gross, Rutledge, Wolf, and Bomblat, 1997). Gross et al. (1997) noted that in systems with poorly functioning or failed septic tanks that do not remove suspended solids; systems may need to be cleaned out as often as twice a year. Effluent filters can be specified to reduce these problems and decrease debris clogging pipes and orifices (Glotfelty et al., 1997).

2. Turn-ups at the distal end of laterals are prone to breakage. This was the most common problem in one follow-up study of installed pressure-distribution systems (Glotfelty et al, 1997). Turn-ups allow easy cleaning of laterals, but clearly increase the opportunity for damage to the piping system. Some designers sleeve end caps, turn-ups, or entire laterals with a larger (e.g. 4" pipe) protect the pipes from other damage. North Carolina (1999) and Delaware (Brown et al., 1991), for example, require turn-ups and require that those turn-ups be sleeved with larger diameter pipe.

3. Backpressure from the aggregate (or "rock binding"), upon which distribution laterals rest, decreases the lateral discharge below that calculated from the orifice discharge formula. The decrease below the expected discharge ranges from 13.7% (Glotfelty et al., 1997) to 17% (Amoozegar et al., 1994). Glotfelty reports that smaller orifices are more likely to be blocked. Orifice shields (Washington State Department of Health, 1999) can reduce this problem, as can placing sleeves on end-caps and turn-ups in larger diameter pipe, as discussed above.

4. Broken pipes, float controls, electrical connections, and improper values all occur in some systems (Glotfelty et al., 1997).

5. Pumps can fail when off switches fail to activate and the pumps run without load or for extended times. Redundant-off pump switches can be used to avoid the risk of pump failure when a switch faults (Washington State Department of Health, 1999).

Experience with and concerns about clogging in pressurized systems has led some designers, regulators, and contractors to be concerned about very small diameter orifices. The low pressure used in pressurized systems can be inadequate to scour small amounts of foreign matter, such as leaves and plastic burrs, from very small orifices. Cogger and Rubin (1983) reported that clogging of distribution systems has been a problem on some LPP systems. Large orifice sizing and high pressure results in greater pipe flows, friction losses, and construction costs.

Figure 2 presents the typical lateral layout for an LPP system. The design, as shown, allows the laterals to drain back to a pumping chamber, thereby avoiding laterals from draining out after the pump turns off. This design also allows orifices to be turned either up or down. In general orifices may face up or down, but must be turned down if laterals cannot drain back to the pumping chamber between doses in areas with potential freezing.

**Figure 2:** Low-Pressure Pipe System (LPP) Details. (After, Feiden et al, 1988)

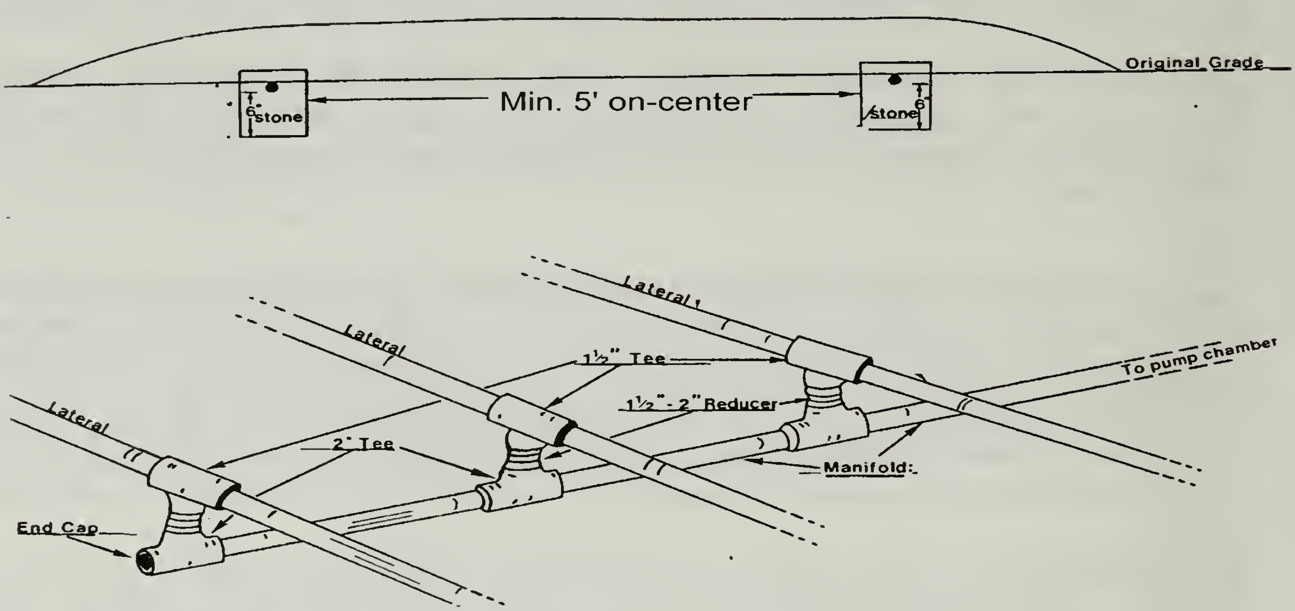


Table 3 below presents a listing of orifice size as reported in the literature and in selected state regulations. Orifice are allowed as small as 3/32" in size, and no state requires orifices larger than ¼" (except for siphon-fed systems). Most states and researchers balance the need for small orifices (which allow lower flows, reduce friction, and make it easier to balance flows to maintain uniform distribution) with concern about clogging by allowing smaller orifices than would be ideal for at least a portion of a distribution network.

**Table 3.** Minimum orifice size and typical ranges in pressure-distributed laterals.

| | **Minimum orifice** | **Typical orifice size range** |
|---|---|---|
| **Research papers:** | | |
| Cogger et al. (1982) | 3/32" | 3/32" to 7/32"<br>5/32" as basis for design |
| Cogger and Rubin (1983) | 5/32" | |
| **State Regulations** | | |
| Delaware (Brown et al., 1991) | 5/32" | 5/32" to 7/32" |
| Maryland (Glotfelty et al., 1997) | 5/16" | |
| Massachusetts (Mass. DEP, 1995) | 1/4" | |
| North Carolina: for LPP systems (NC Env. Health, 1999) | 1/8" for up to 1/3 of holes<br>5/32" for other holes, except:<br>5/32" for all holes for food preparation facilities (due to greases and oils) | |
| Oregon (Oregon, 2000) | 1/8" (5' of head required) | |
| Pennsylvania (Pennsylvania, 1998) | 1/4" and 5/16" (for siphons) | |
| Washington (Washington State Dept. of Health, 1999) | 1/8" (5' of head required) | 3/16" and above (2' of head required) |

Increasing orifice size, pump size, pipe diameter and the number of manifolds can quickly add substantial cost to a system. Increases in the void volume of the pipes, is generally not recommended. At the end of each dose, when the pump turns off, the void volume generally either drains back to the pumping chamber, requiring increased pumping, or drains to the system by gravity, and therefore is discharged of without uniform distribution.

In southern states and coastal areas, void volume draining back to the pumping chamber is typically avoided by using a check valve (e.g. Oregon, 2000). In Massachusetts and areas with frost penetration, however, maintaining water in pipes between doses through the use of check valves is often avoided to avoid frost problems. Check valves which maintain water between doses can be used in Massachusetts by placing the delivery pipe below frost line. This adds to cost and increases the danger of ground water infiltration, however. Because of the desire to avoid backpressure on pumps and inadvertent siphoning of effluent out of a pumping chamber, check valves are typically used even in cold climates. Check valves are typically used in cold climates with a weep hole drilled in the force main immediately up gradient of the check valve.

Even with check values, distribution laterals with orifices facing downward, or with orifices facing up if the laterals are not all at the same elevation, will drain a portion of the dose by gravity at the end of each dose. In a well-designed system with level laterals and adequately spaced orifices, there is relatively little flow through the laterals at the end of each dose, and therefore a majority of the effluent is distributed uniformly.

Pressure distribution systems in warmer climates typically have holes facing up (e.g. Oregon, 2000). This minimizes problems with the draining of the void volume of the pipe by gravity after the pump turns off. Unfortunately, however, in cold climates this can create freezing problems if the pipes do not drain back to the pumping after each dose and are remain full of water. For example, Oregon (2000), while assuming that lateral orifices will face up, allows holes to face down when frost would otherwise create problems.

Berkowitz (1985) presents a model for pressure manifold design for conventional drain fields and LPP drain fields that ensures equal distribution to each lateral, although not necessarily within each lateral. On a site without equal distribution within laterals, the more laterals that were used the more uniform the flow would be over the entire system. This design was especially appealing because it reduced the need to do iterative calculations on flow within each lateral.

Winkler and Feiden (1997) developed the Pressure System Distribution Spreadsheet (PSDS) for simple to complex LPP designs. This robust application allows up to 26 laterals and 50 orifices per lateral. Provisions are made for design on sloping sites, unequal lateral lengths, complex manifold systems and orifice sizing. Originally designed to provide rapid verification of small low-pressure pipe system design the PSDS program is capable of meeting the design requirements for a range of small to large-scale ST-LPP systems. Winkler and Feiden (1997) noted in their work that Otis's head loss computation (Otis, 1981) is designed primarily for small systems, less than 600 GPD. For systems larger than 600 GPD, Otis tends to underestimate the pressure head losses even with safety factors specified by Otis (1981).

Otis's seminal article (1981), upon which *Massachusetts Title 5 Appendix: Guidance for Pressure Dosing Systems* (Mass. DEP, 1985) is based, calls for a minimum of 2.5 feet of pressure. Otis cautions, however, that excessive pressure adversely affects orifice discharge rate and network losses, because of resulting friction losses. The smaller the orifice size, the more pressure that is needed to ensure that effluent flow scours the orifice and minimizes the opportunity for orifices to clog.

Table 4 presents a listing of minimum distal pressures from the literature and selected state regulations. These range from 2 to 5 feet, with most in the 3 to 4 foot range. Not surprisingly, because higher-pressure systems have less clogging, the states allowing the smallest orifices either require the higher head (e.g. Delaware) or higher head when small orifices are used (e.g. Oregon).

Some design guidelines allow a 25% difference in discharge rates between orifices in the system (Cogger et al., 1982). Although LPPs are probably robust enough that a 25% difference may still provide adequate effluent distribution, the use of computer spreadsheet applications (Winkler and Feiden, 1997), even when designing for large systems on sloping sites, allows for much tighter tolerances in regulatory and design standards.

**Table 4.** Minimum distal pressure within pressure-distributed laterals

| | Minimum distal pressure (feet) | Typical range (in feet) |
|---|---|---|
| **Research papers:** | | |
| Spooner et al. (1998) | 4' | 4'-8' |
| Cogger and Rubin (1983) | 3' | |
| Otis (1981) | 2.5' | |
| **State regulations:** | | |
| Delaware (Brown at al., 1991) | 4.6' | 4.6' to 6.9' (5.8' recommended |
| Maryland (Glotfelty et al., 1997) | 2' | . |
| Massachusetts (Mass. DEP, 1995) | 2' | 2' to 3' |
| North Carolina: for LPPs (NC Env. Health, 1999) | | 2' to 5' |
| Oregon (Oregon, 2000) | 5' | 5' |
| Pennsylvania (Pennsylvania, 1998) | 3' | |
| Washington (Washington State Dept. of Health, 1999) | 2' | 2' to 5' (depending on hole size) |

North Carolina, which specifically allows LPPs, and Pennsylvania and Washington, which allow related systems, (NC Environmental Health, 1999; Pennsylvania, 1998; and Washington State Department of Health, 1999) all require that discharge rates on the same lateral have no more than a 10% difference, and that discharge rates on all laterals within a system have no more than a 15% difference. Oregon's regulations (Oregon, 2000) require that all flow rates vary by no more than 10%. Oregon also requires 2" laterals, which reduce friction, thereby allowing more even flow under pressure. However, larger laterals also increase void volume, which increases the percentage of the dose that drains by gravity instead of under full-pipe flow pressure.

There are indications that systems receiving frequent small doses provide more effective denitrification than systems with occasional large doses. (Harkin et al., 1979). Doses must still be large enough to effectively use the entire soil absorption field, including the area between orifices, and to minimize pipe void volume as a percent of the total dose. More frequent doses, unfortunately, reduce the resting time between doses and contribute to soil clogging (Otis, 1985).

Oregon (2000) requires a <u>maximum</u> dose of 20% of daily design flows. This allows for relatively small doses, but compared to more common dosing recommendations of 1, 2 or 4 times daily (Mass. DEP, 1995; Brown et al., 1991;Otis, 1981; Otis, 1985; Washington State Dept. of Health, 1999), increases pipe void volume as a percent of the total dose. The void volume problem is minimized by Oregon's practice of placing the lateral orifices facing up. Orifices facing up can cause pipe freezing in cold climates if laterals do not gravity drain completely back to the pumping chamber between doses.

For uniform distribution to work properly, void volume must be kept to a minimum. This true for all systems, but is especially true in cold climates sites where it is not practical for laterals to drain back, such as, when a lateral is lower in elevation than the pumping chamber or than any point on the force main. In all pressurized pipe systems, pipe drain freely while the laterals are filling and after the pump turns off (Amoozegar et al., 1994). Reduced void volume in the laterals will minimize the portion of flow not applied under full pipe flow pressure.

## Conclusions

Shallow-Trench Low-Pressure Pipe Wastewater Treatment Systems (ST-LPPs) are pressurized, shallow-placed, narrow-trenched soil absorption systems. Trenches can be placed as shallow as the A horizon, consist of as little as 8" of stone, and be as narrow as 9" wide. Effluent is dosed and uniformly distributed through small diameter (1.5 to 3 inch) pipes under low pressure (typically 2.5 to 4 feet of head) for small to medium sized systems.

When properly designed and sited, ST-LPPs can be effective on most sites where conventional systems are appropriate. They can also overcome some site limitations that require site modification or some types of alternative systems. Their shallow placement alone provides greater vertical separation from groundwater, ledge, or other limiting layers than conventional systems. ST-LPPs may be suitable for sites where mounded systems are required. However, they are not suited for every site or every site limitation.

Because ST-LPPs utilize the A and B soil horizons, where biological treatment is the greatest, they can provide treatment enhancements over conventional systems, especially in treating biological contaminants. The combination of shallow placed and narrow trenches maximizes interface with the native biologically active soil layers. A limited set of data is available on reduction of nutrients, nitrogen and phosphorus. ST-LPPs have not been demonstrated to provide any significant enhancement for nitrogen reduction as compared to conventional systems. Uniform pressure distribution has been demonstrated to enhance hydraulic performance on sites with moderate to severe soil limitations. ST-LPPs can also work effectively on sites with surficial site constraints, such as steep slopes, rocks, and specimen trees. The use of pressure-distributed allows the shallow narrow trenches to be laid out following the contours and curving around surficial site constraints. Construction of ST-LPPs is relatively simple as compared to conventional gravel bed soil absorption systems and may be less expensive than other mounded pressurized systems

Proper design, siting, and installation are critical to ensure appropriate LPP performance, function, and longevity. Design guidance is available in the appendix of this document and can be referenced from other sources.

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# APPENDIX A: DESIGN GUIDANCE

## Shallow-Trench Low-Pressure Pipe Systems

The literature review on Shallow-Trench Low-Pressure Pipe systems (ST-LPPs) suggests that properly designed ST-LPPs can potentially be more effective than both gravity and dosed conventional systems at removing some or all wastewater contaminants[1]. In addition, when designed properly, ST-LPPs are capable of solids reduction and performing hydraulically in most climatic conditions, even on sites with moderate to severe limitations. Shallow-trench LPPs are notably less expensive than sand filled pressurized mound systems and may be easier and quicker to install. Numerous states throughout the US have codified use of pressure distribution systems placed in the surface horizons either as conventional or alternative technology. The permitting of these systems may be hastened in Massachusetts using a design guidance based on "State-of-the-Art" practice. This design guidance is based on analysis of the available information on ST-LPPs and LPPs in general.

Low-pressure pipe wastewater treatment systems, especially shallow-trench LPPs, provide several major benefits over conventional systems:

1. The use of pressure distribution clearly provides uniform distribution of effluent throughout the system and prevents or minimizes short-circuiting of effluent out of the first few holes and into the groundwater. This is especially important in sandy soils and in stony soils, where short-circuiting provides the greatest threat to groundwater. As a result, the use of pressure distribution/uniform flow of any kind provides the greatest benefit on these soils.

2. In a conventional system, the extra foot depth to groundwater required in Massachusetts for excessively fast percolating soils provides limited benefit. The extra foot of separation provides a theoretical increase of 24 minutes residency time. This assumes saturated flow at 2 minutes per inch over 12 inches of depth. Even if partially unsaturated flow is assumed with an order of magnitude reduction in flux, the added residency time is only 6 hours. The benefit of pressure-distribution/uniform flow, with LPPs or any kind of allowed system, is far more significant than this extra depth because it spreads effluent out over the entire soil absorption field and increases the average residence time of effluent in unsaturated soil. As such, pressure distribution/uniform flow is probably significantly more effective than gravity distribution and an extra foot of vertical separation.

3. The use of shallow-trench systems provides greater separation to groundwater, ledge, and other soil limiting layers, which provides greater unsaturated soil depth for treatment than in a

[1] The available research in the literature does not demonstrate that shallow-trench low-pressure pipe systems are any more effective at removing nitrogen than conventional systems. At this time, based on the lack of data to support nitrogen credits, ST-LPP systems should not receive any extra nitrogen removal credit for nitrogen removal.

conventional subsurface soil absorption system. Shallow-trench LPP systems also provide for greater native soil interaction than in a Massachusetts' sand fill system. Shallow-trench LPPs can provide increased treatment as compared to systems built in fill, since fill material generally does not provide as effective treatment as native soil.

4. LPPs are effective hydraulically, even in very tight soils and in the A and B soil horizons. By design, removal of the topsoil and subsoil (A and B soil horizons) is not required for an LPP system to function properly. These shallow placed systems take full advantage of the biologically active upper horizons.

## Design Methods

The following steps and design methods should be used to design a ST-LPP system.

**1. Determine daily design flow based on the same regulatory standards as used for conventional systems.**

**2. Size the septic tank based on regulatory standards for conventional systems.**

**3. Design pumping chamber based on Massachusetts regulatory standards for pumped systems.**

**Approach:**

a. Pumping chamber must be sized to store a minimum of one day of design flow above the alarm switch. The pumping chamber must have a manhole access that is accessible at the ground surface.
b. Pumping chamber must use mercury, or other highly reliable, float on and off switches set to dose 3 times a day. The dose volume should be set at 33% of daily design flows, plus effluent which drains out of a pumping chamber weep hole during pumping, plus effluent that drains back to the pumping chamber after the pump turns off at the end of each dose. Variable dose volumes must be considered to deal with site or design specific issues.
c. Pumping chamber must have an alarm switch just above the pump on switch, wired back to the principal building with visual and auditory alarms.
d. Pump cycle counters are highly desirable to allow monitoring of system performance.
e. The sewage effluent pump(s) must meet Massachusetts' standards, including dual alternating pumps required for all but single and two-dwelling unit systems.
f. Under Massachusetts Environmental code, 310CMR 15.233, use of siphons for on-site sewage disposal systems is prohibited unless approved as a component of a recirculating sand filter or other alternative.

**Rationale:** The pumping chamber is no longer considered alternative technology and there is little unique about the needs of a pumping chamber for an LPP system. Dosing and pressure

distribution ensures uniform flow and alternating systems of dosing and resting. This provides the improved treatment benefits of uniform flow, including preventing short-circuiting of effluent. Dosing three times a day (instead of less frequently) reduces the need for storage of larger flows in the trenches. More frequent doses may increase denitrification and phosphorus removal. However, dosing more frequently may create problems with the larger percent of a dose that is not under pressure before full pipe flow and after the pump turns off and shorten pump life.

## 4. Design trench width:

**Approach:** Trenches should have a minimum width of 9" and a maximum width of 3'. The preferred width is 12". Installation should be specified using a continuous digging trencher or small backhoes with rakes are preferred over large backhoes. Large backhoes should be avoided if at all possible. All surfaces of the trenches should be scarified.

**Rationale:** Narrow trenches provide the best treatment by maximizing interface with native soil. A trench width of 9" is wide enough to allow construction with a continuous digging trencher (a ditch witch), which works well on sites with no rocks and provides all of the benefits of very narrow trenches. A trench width of 12" can provide most of the benefits of narrow trenches while easing the construction burden (very narrow backhoe buckets and can be used) and provides slightly more dose storage and a slightly lower loading rate. Given the need to have relatively few doses, 12" trenches are a good balance of narrowness and enough volume to store a large effluent dose.

The maximum width allows conventional backhoes to be used, but does not provide a reduction in linear loading rates of trenches because as trenches get wider, there is less contact with native soil and more potential for groundwater mounding. In addition, large backhoes disturb the soil more and should be discouraged except where they are the only practical option. The performance advantage of narrow trenches suggests that no loading credit should be given for trenches wider than 12".

## 5. Design trench depth:

**Approach:** Trench depth should be a minimum of 6" to a maximum of 9" below the lateral pipe invert. Lateral pipes should be covered with 2" pea stone above and around the pipe.

**Rationale:** Trench depth and width should be as shallow and narrow as possible, but with enough void volume to absorb each dose. Shallow trenches maximize vertical separation from limiting layers. Shallow and narrow trenches are preferable to systems, which simply utilize hydraulically head in the trench to drive effluent into the groundwater. The large surface area contact in narrow trenches increases the potential for using natural capillarity to drive effluent into the subsurface, resulting in longer retention times and greater treatment. Less pea stone is

needed above the lateral pipe because pressurized distribution pipe will be smaller in diameter than that used in a conventional gravity system.

### 6. Determine total trench length required using the loading rates given below.

**Approach, Alternative One:** Design flow divided by the loading rate in Table A1, below, determines how many linear feet of trenches are required.

**Rationale:** Loading rates are based on current Massachusetts' regulations for trenches, with credit for trench bottom and sidewall space. These rates assume 9" wide trenches and 6" trench sidewalls. A minor credit is also given because of the improved treatment obtained by shallow placement, uniform distribution, and narrow trenches. No credit is given for trenches deeper than 6" (below the pipe invert) or wider than 12". The table is also simplified and rationalized for maximum treatment by being based on texture and not percolation rate. Foreseeable changes to soil based loading rates could easily be mirrored in LPP loading rates.

--Class I sands and loamy sands: Short-circuiting is significantly less problematic in uniform-distributed systems versus conventional systems, therefore sands could be given a large credit to increase the allowable loading rate. Instead, due to shorter retention times it is recommended that no credit (relative to Title 5 for conventional systems) be given for fast percolating sandy soils, because of concern about short-circuiting and rapid movement of wastewater through coarse textured soils. Only a small credit should be given for other sands. Treatment in sands can be improved by ensuring uniform distribution over the entire soil absorption area. In addition, giving a credit in these soils provides little cost savings.

--Class II sandy loams and loams: Maximum credit ranges is approximately 50% for the slowest loams, to reflect the benefits of LPPs and the findings in the literature, to none for the highly permeable sandy loams, to reflect the concerns described above

--Class III silt loams: Credit ranges up to approximately 50% for silt loams, to reflect the benefits of LPPs and uniform distribution.

--Class IV clays and silty clay loams: No credit given, because of low permeability and to reflect the more limited benefits of uniform distribution in very slowly permeable soils. Generally, uniform distribution provides less benefits on very fine textured and dense soils, although the LPPs ability to spread effluent uniformly throughout especially long pipe runs in shallow and narrow trenches ensures that LPPs are capable of working in these soils while providing hydraulic and treatment benefits.

**Table A1.** Recommended Loading Rates (in gallons per day per linear foot of trench)

| Title 5 CLASS | CLASS I | CLASS II | CLASS III | CLASS IV |
|---|---|---|---|---|
| Soil texture | Sands/ Loamy sands | Sandy loams/ Loams | Silt loams | Clays/Silty clay loams |
| Linear loading rate for 9" wide trench[†] | 1.3[*] | 1.05 | 0.65 | 0.25 |
| Linear loading rate for 12" wide trench[†] | 1.4 | 1.2 | 0.74 | 0.3 |

[*]Design flow/loading rate = Total required trench length

[†]Assumes minimum trench spacing of 5' on center and 6" sidewall invert depth

**Approach, Alternative Two:** Design flow divided by the Title 5 loading rate, based on 9" wide trenches and 6" effective trench sidewalls, determines how many linear feet of trenches are required with no additional credit given for wider or deeper trenches).

**Rationale:** Loading rates are Massachusetts' regulations for trenches. No credit is given for trenches deeper than 6" (below the pipe invert) or wider than 12".

## 7. Determine number of trenches and their layout.

**Approach:** Minimum separation between trenches 5' on-center, but no less than 4' of undisturbed area between trenches. There is no maximum trench length or minimum or maximum number of laterals, provided that the designer demonstrates that pressure distribution meets uniform flow standards provided below. There is no maximum separation between trenches, and much larger separations will often be necessary to allow trenches to avoid rocks, trees and other obstacles. Each trench shall follow the contour of the land and each trench shall be level. Trenches can be of different elevations from each other, so long as uniform flow standards are met.

**Rationale:** Five-foot on-center placement (minimum four feet separation for wider trenches) minimizes groundwater mounding, provides dilution, allows lateral movement of effluent into trench sidewalls and through the soil profile, and preserves an adequately sized replacement area. Level trenches prevent effluent from flowing by gravity during a dosing cycle. This recommended separation is the basic design used in other states' regulations and LPP design manuals, and the positive research on LPPs is based on this standard design. There is inadequate research to support a recommendation for less separation at this time.

## 8. Depth to impervious layer, groundwater and ledge:

**Approach:** Systems should be constructed in the A horizon and/or B soil horizons. When possible, LPPs should be laid out to maintain the same vertical separation to impervious layers,

groundwater, and ledge as required for conventional systems. A 12-inch reduction in depth to limiting impervious layers or seasonally high groundwater may be justified when using a ST-LPP due to improved treatment potential and reduction of effluent short-circuiting. In systems with design flows greater than 2,000 GPD, reductions in depth to groundwater should be verified using mounding analysis tools to determine that subsurface mounding will not further reduce depth to groundwater.

**Rationale:** A 12" credit in depth to groundwater for ST-LPP systems is possible for several reasons: 1) Equal distribution reduces short-circuiting into the soil profile. 2) Equal distribution prevents excess amounts of effluent from concentrating in one area, especially in soils with high permeability. 3) Narrow trenches with adequate horizontal separation reduce ground water mounding. 4) ST-LPP can provide greater water quality treatment than conventional systems.

While the literature is not conclusive with regard to reduced depth requirements, shallow placed pressurized systems have been demonstrated to provide equal or better hydraulic performance with reduced isolation in some studies. Additional work in this area is warranted

## 9. Design for future maintenance:

**Approach:**

a. End caps on all laterals should include turn-ups to bring the pipe just below grade for maintenance and periodic cleaning.
b. Turn-ups should be capped in a 4" Schedule 40 (SCH 40), 4" SDR 35, or a thicker or more durable pipe sleeve with an easily removable inspection end-cap at grade.
c. All pipes, and especially turn-ups, should be located on as-built plans or tie-cards.
d. Use effluent filter before pump to filter small solids and reduce the potential for clogging[1]
e. The use of redundant-off pump switches, to avoid the risk of burning out the pump if switch fails, is recommended.

**Rationale:** If the system ever clogs, it is important to be able determine the exact location of the turn-ups or end caps. Turn-ups tend to get broken during lawn maintenance or by homeowners who don't want to see the pipes. Therefore the distal orifice should be buried and protected with durable pipes or sleeves. The final orifice in the end caps or turn-ups, which allow air to escape when the network pressurizes, must be protected to keep back aggregate and allow free discharge.

[1] Use of effluent filters is permitted under Title 5 (MA DEP, 1995) and may be effective in reducing solids loading to the soil absorption field. However, effluent filters require significant maintenance, without which septic tank system failure is probable.

## 10. Design for pressure distribution and uniform flow.

### A. Determine first approximation of orifice size and spacing.

**Approach:** Calculate the head losses and orifice discharge from a first cut lateral layout, orifice size and spacing. Discharge should be calculated using a minimum orifice size of ¼" using 2.5' to 3' minimum distal pressure. Smaller orifice size (5/32") may be allowed when justified, provided that greater distal pressure is used or when needed for a relatively small number of orifices to balance the system. Orifices should be spaced between 36" and 48". Maximum spacing should be 48" for sands and 60" for other soils. Alternate orifice spacing is acceptable where it is to balance flows, especially where laterals are different lengths and at different elevations. Orifice size and spacing will need to be adjusted as pressure calculations (below) are performed.

Orifices are generally designed to face down in the soil absorption field (except for the end cap). Orifices can be used facing up, with orifice shields to keep stone from blocking the orifices, if the soil absorption field is designed to drain back to the pumping chamber after each dose and if fittings will not trap any water that can freeze.

**Rationale:** Pressure distribution is designed to ensure uniform flow, alternating the process of dosing and resting, and prevent short-circuiting of effluent. Orifices must be large enough to prevent clogging. Designing for greater distal pressure can minimize clogging in smaller orifices. No system should be designed with distal pressure less than 2.5 feet of head. Increase pressure and flow velocity not only clears orifices of solids but also provides scouring of distribution pipe preventing solids buildup. Orifice spacing must be far enough apart to allow uniform flow, without being so far apart that parts of the soil absorption field do not receive effluent. Orifice spacing is more important on sands and coarse textured soils, where there is a greater risk of short-circuiting into the groundwater

Except in coarse textured sandy soils, orifice spaced at 48" is generally adequate to provide uniform distribution along the length of a trench. Narrower trenches could have wider spacing, as the dose will extend further through the void space of the aggregate. Orifice spacing, or number of orifice per lateral length must be balanced to provide uniform flow and minimize frictional losses. A system designed with excessive discharge orifices will require increased pump rates and higher pressure capacity. If designing orifice spacing based area, a minimum of one orifice per 25 $ft^2$ of basal area can be recommend for uniform distribution and minimizing pump size.

Discharge orifice can be oriented facing down or up. In New England, downward facing orifice is the standard practice. The orientation in the downward direction ensures that discharge laterals drain completely between doses and minimizes freezing potential. In some cases where drainage back to the pumping chamber is not possible, orifice must be facing downward. Orifice oriented in the upward direction minimizes the portion of each dose, which is discharged under partial pressure or gravity at the beginning and the end of each pump cycle. Where drainage back to the pumping chamber is assured, orifice can be place in the upward orientation. In either position,

orifice must be protected from blockage from aggregate. Use of orifice shields or lateral sleeves may be used.

## B. Determine appropriate pipe size.

**Approach:** A table pipe diameters with maximum flow before frictional losses become none laminar is presented in Table A2. These values are based on Otis (1981) and are approximate limits. Pipe will carry much higher flow rates but at greatly increased frictional losses, which are difficult to model.

1. Pressure pipe is specified in Title 5 (MA DEP 1995) and should be SDR 35 or Schedule 40 (C=150).
2. Pressure pipe for effluent distribution or laterals require drilled orifices to size.
3. The distal orifices should be drilled near the top of each end cap or turn-up approximately three quarters up from the invert of the pipe.
4. The force main or delivery pipe (connecting the pumping chamber to the system) and the manifold (connecting the force main with all of the laterals) should be solid pipe. A weep hole may be placed in the manifold to allow it to drain after each dose if it is larger in diameter than the discharge laterals. Large systems may use telescoping manifolds.

**Table A2.** Minimum Recommended Pipe Size for Pressure Distribution

| Pipe Size | Maximum Flow before friction losses become excessive (after Otis, 1981) | Application Notes |
|---|---|---|
| 1.5" | 25 gallons per minute | Most common lateral size for small* systems |
| 2" | 50 gallons per minute | Most common manifold size. Delivery pipe for small systems. Lateral size for medium to large systems. |
| 3" | 150 gallons per minute | Delivery pipes for small and medium systems. Laterals and manifolds for medium to large systems. |
| 4" | 300 gallons per minute | Delivery pipes for medium to large systems. Laterals and/or manifolds for very large systems. |
| 5" | 900 gallons per minute | For very large systems. |

* Small systems <600 GPD
Medium systems 600 to 2,000 GPD
Large systems 2,000 to 20,000 GPD

**Rationale:** Pressure pipe must be sized to minimize frictional losses. Larger pipe will generate lower friction losses on constant flow. Oversized pipe should be avoided because they have more void volume increasing pumping requirements. In particular, lateral pipe size must be minimized such that at the beginning and end of each pump cycle the void volume of the pipe draining under partial pressure is minimized. General guidance suggests that the void volume of the discharge laterals be less than 5 to 10% of the design dose volume. The smaller this value is the greater likelihood of the pressure system performing as designed. Flow through smaller pipe results in greater frictional head loss resulting in excessive differences in discharge rates and larger pump

requirements. Pipe sizing must be balanced to achieve uniform orifice discharge and effluent distribution and minimize pump requirements.

### C. Calculate head and friction.

**Approach:** The pump must be sized to operate at pressure heads to that exceed or equal static head loss or the elevation difference between the pump-off switch and highest point in system, distal pressure, pipe frictional head loss, and entry head loses. If orifices smaller than ¼" are used the distal pressure should be increased to a minimum of 4 to 5 feet of head. The Hazen-Williams Formula is used to determine frictional losses along pipe lengths.

$$F = L_d \, [3.55 \, Q_m / C_h \, D_d^{2.63}]^{1.85} \quad \text{(Hazen-Williams, after Otis, 1981)}$$

Where:

$F$ = Friction Loss in feet
$L_d$ = length of pipe in feet
$Q_m$ = discharge rate in gallons per minute
$C_h$ = roughness coefficient (150 for plastic pipe)
$D_d$ = pipe diameter in inches

Pipe fitting and entry head losses (energy loss as fluid enter orifice), are small enough such that adding a safety factor of 15% to the total system head loss is appropriate. If other than standard fittings are used, actual fitting head loss can be added.

**Rationale:** Accurate calculation of friction losses must be made to assure uniform distribution. Pump sizing is based solely on the predicted flow rate in GPM and frictional loss. The process of calculating friction losses involves iteratively adjusting pipe size, orifice size, orifice spacing, lateral length, and lateral and manifold configuration. As an iterative process the best computation method for accurately predicting friction losses is using computer software designed for such purpose (Winkler and Feiden, 1997). Greater head is required when using small diameter orifices to prevent clogging of orifices. Friction calculations must include fittings and entry head losses, which may be approximated as 1 foot or 1/3 of network head losses. A safety factor may be applied to the entire system to approximate theses loses..

### D. Orifice discharge formula and orifice discharge uniformity check.

**Approach:** For more complex systems utilizing non-symmetrical laterals or differing orifice sizes, differences may be based on soil-absorption area rather than orifice differences. The design should have no more than 10 to 15% difference in orifice discharge anywhere in the system. This value allows a safety margin for field conditions.

Orifice size and spacing must be adjusted to maintain equal distribution. The orifices should never be less than 5/32" (to avoid clogging), although ¼" is preferred when possible. Orifices should not be spaced further apart than six feet on-center.

Use Orifice Discharge Formula (derived from $Q = CA(2GH)^{1/2}$) for each orifice and lateral segment in the system:

$$Q_m = 11.79BD^2H_d^{1/2}$$

Where:

$Q_m$ = Discharge in Gallons per minute

D = Diameter of orifice in inches

$H_d$ = Head in feet, 2.31' (1 PSI) to 3.0'

B = Correction for backpressure from aggregate on pipe orifice. (0.8 to 1.0 with orifice shields or gravelless systems).

**Rationale:** Conditions in the field are never exactly what the design shows, so a margin of safety should be built into the design. Orifice spacing would ideally be 5/16" holes every 2.5 or 3 feet, but friction and equal distribution becomes more difficult with larger orifices and more frequent spacing.

Orifice discharge assumes free discharge, not the backpressure of stone (stone or rock blockage) against the laterals. To compensate, either orifice shields or gravelless systems can be used, or the backpressure component can be adjusted to assume that flow will be 80% to 90% of what would otherwise be expected with free discharge. Some older systems were designed with orifice discharge differences as great as 25%. The wide use of computational tools such as computer spreadsheets (Winkler and Feiden, 1997) simplify data manipulation and accuracy of calculations. Rapid design calculations through manipulation of hole sizes and spacing allow much greater system control and hence orifice discharge differences less than 15% are reasonably attainable. On sloping sites or where laterals must be different lengths, other measures of uniformity must be used. A typical approach is to match total discharge per entire lateral length for uniform length laterals at different elevations or match discharge per square foot of trench length for systems with laterals of different length and elevation. These alternative methods produce the same net effect of uniform orifice distribution, system wide uniform distribution, but minimize system complexity with uniform orifice sizing. Due to the potential for variation in manually drilled hole, use of a single orifice size is recommended.

## 11. Final system inspections and check of all system components.

**Approach:**

a. Inspect septic tank, pumping chamber, pumps, switches electrical connections, dose levels, and alarms.
b. Check that design layout matches design.
c. Inspect discharge orifices for burrs and clean out the burrs.

d. Inspect pipe end caps or turn-ups and ensure they are properly sleeved, supported by stone, and end just below or at the ground surface.
e. Test for specified pressure and uniform distribution across system.
f. Prepare as-built plans or tie-cards showing all system components, especially turn-ups.
g. Prepare clear owners' operation and maintenance manual for system, explaining use and maintenance of system.

**Rationale:** Poor design and construction create many problems reported as system failures. For example, broken turn-ups are the most commonly reported problem in system performance, yet this problem can be easily avoided. System installation supervision is critical for successful use and operation of most innovative technologies.

## Design Checklist

| DESIGN STEP | | FILL IN OR CHECK BLANKS |
|---|---|---|
| 1 | Determine daily design flow based Title 5 standards for conventional systems | _____Gallons Daily Design Flow |
| 2 | Size the septic tank(s) based on Title 5 standards for conventional systems. | _____Gallon Septic Tank (s)<br>_____2 tanks/compartments required |
| 3 | Design pumping chamber based on Title 5 standards for pumped systems. (With one-day design flow above alarm switch level) | __________Chamber size or dimensions<br>_______1-day storage above alarm switch,<br>_______Mercury switches and alarm |
| 4 | Design trench width | _______9" wide<br>_______12" wide<br>_______" wide (use 12" linear loading rate) |
| 5 | Design trench depth<br>Trench bottom shall be a minimum of 9" below grade. | _______9" trench (6" of stone below pipe)<br>_______12" trench (9" of stone below pipe)<br>_______" from grade to bottom of trench |
| 6 | Determine minimum total trench length using the loading rates given in Table 5 in design guidance. | _______linear loading rate<br>_______design flow/_____loading rate=<br>_______total trench length |
| 7 | Determine number of trenches and their layout. (Trenches do not have to be symmetrical. They should follow contours) | _______number of trenches<br>_______length (average) per trench<br>_______trenches on-center (minimum 5') |
| 8 | Depth to impervious layer, groundwater and ledge (Title 5 applies, except for reduction in depth to groundwater in fast percolating soils and possibly piloting reduction for other sites.) | _______depth to impervious layer<br>_______depth to ledge<br>_______depth to groundwater<br>_______request reduction to groundwater |
| 9 | Design details | _______capped turn-ups to below grade<br>_______as-built plans or tie-cards<br>_______effluent filter<br>_______redundant-off pump switch |
| 10 | Design for pressure distribution and uniform flow, based on Title 5 and this guideline. (System does not have to be symmetrical. Detailed design and spreadsheet must show full details of system layout.) | _______force main (2" minimum)<br>_______manifold (2" minimum)<br>_______laterals (1.5" minimum)<br>_______orifice size (.25" minimum)<br>_______orifice spacing<br>_______distal pressure (2.5' minimum)<br>_______orifices up or down<br>_______total friction headloss<br>_______percent difference between orifices<br>_______detailed design spreadsheet attached |

## Design Example: Required Spreadsheet

| | |
|---|---|
| DESIGN FLOW (in gallons/day)? | 440 |
| Elevation of the PUMP OFF SWITCH, in feet? | 100 |
| Elevation of the upper LATERAL, in feet? | 105 |
| DELIVERY PIPE distance, from pump to manifold, in feet? | 100 |
| DELIVERY PIPE diameter, in inches (if not 2"--use 2" min)? | 3 |
| Design DISTAL PRESSURE, in feet (if not 2.5)? (hd) | 3 |
| IS MANIFOLD CENTER-FED & SYMETRICAL (yes or no)? | yes |
| How many orifices in the MANIFOLD? | 0 |
| MANIFOLD ORIFICE diameter, in inches (if not 5/16") | 0 |
| MANIFOLD DIAMETER (if not 2"--use 2" min)? | 2 |
| TOTAL LENGTH OF MANIFOLD | 6 |
| Does MANIFOLD drain to FIELD after dose (yes or no)? | no |
| How many LATERALS? | 4 |
| Pumping chamber weep hole size (usually .25") | 0.25 |

| (first orifice from lateral 1/2 of orifice spacing) | Lateral 1: | Lateral 2: | Lateral 3: | Lateral 4: |
|---|---|---|---|---|
| Length of each LATERAL, in feet? | 90 | 90 | 90 | 90 |
| Diameter of each LATERAL, in inches (1.5" min)? | 1.5 | 1.5 | 1.5 | 1.5 |
| Elevation of each LATERAL, in feet? | 105 | 105 | 104 | 104 |
| Number of ORIFICES per lateral | 26 | 26 | 23 | 23 |
| Distance from Manifold to closest Orifice, in feet | 2.5 | 2.5 | 2 | 2 |
| ORIFICE SPACING, in feet | 3.5 | 3.5 | 4 | 4 |
| Diameter of ORIFICES, in inches? (D) | 0.25 | 0.25 | 0.25 | 0.25 |
| Square feet of leachfield per laterals (can ignore) | 92 | 92 | 92 | 92 |

| | |
|---|---|
| Maximum number of orifices in any one lateral | 26 |
| Minimum lateral diameter | 1.5 |

**RESULTS**

FRICTION CALCULATIONS (using Hazen Williams friction ft= Ld((3.55Qm/Ch(Dd^2.63)))^1.85)
PRESSURE CALCULATIONS (using orifice dischage equation Q=11.79 D^2 hd^.5

| | Lateral 1: | Lateral 2: | Lateral 3: | Lateral 4: |
|---|---|---|---|---|
| LATERAL DISCHAGE (first approximation) | 33.18 | 33.18 | 29.36 | 29.36 |
| MANIFOLD ORIFICE DISCHARGE | 0.00 | | | |
| TOTAL SYSTEM DISCHAGE (first approximation) | 125.08 | | | |
| TOTAL DISCHARGE PER LATERAL | 33.76 | 33.76 | 34.41 | 34.41 |
| DISCHARGE PER SQUARE FOOT OF LEACHFIELD | 0.36692501 5 | 0.36692501 5 | 0.37406614 | 0.37406614 |
| ORIFICE MAXIMUM DISCHARGE BY LATERAL | 1.34 | 1.34 | 1.54 | 1.54 |
| ORIFICE MINIMUM DISCHARGE BY LATERAL | 1.28 | 1.28 | 1.47 | 1.47 |
| ORIFICE % DIFFERENCE DISCHARGE within LATERAL | 4.6% | 4.6% | 4.1% | 4.1% |
| MAXIMUM DISCHARGE LATERAL | 34.41 | | | |
| MINIMUM DISCHARGE LATERAL | 33.76 | | | |
| MAXIMUM DISCHARGE PER SQUARE FOOT | 0.37 | | | |
| MINIMUM DISCHARGE PER SQUARE FOOT | 0.37 | | | |
| % DIFFERENCE DISCHARGE for SYSTEM by orifice | 17.0% | of maximum orifice in system | | |
| % DIFFERENCE DISCHARGE for SYSTEM by laterals | 1.9% | of maximum lateral in system | | |
| % DIFFERENCE DISCHARGE for SYSTEM by square feet | 1.9% | of maximum square foot in system | | |
| WEEP HOLE DISCHARGE (usually a 1/4" weep hole) | 2.71 | | weep hole= | 0.25 |
| VOID VOLUME IN DELIVERY PIPE | 36.72 | | | |
| VOID VOLUME IN MANIFOLD | 0.98 | | | |
| VOID VOLUME IN EACH LATERAL | 8.26 | 8.26 | 8.26 | 8.26 |
| TOTAL LATERAL VOID VOLUME | 33.05 | | | |
| MINIMUM DOSE VOLUME (based on void volume) | 165.23 | to | 330.46 | MIN |
| ACTUAL MINIMUM IS BASED ON DAILY DESIGN FLOW | (weep hole not counted for dose) | | | |
| TOTAL HEAD LOSS IN EACH LATERAL | 3.29 | 3.29 | 3.42 | 3.42 |
| MAXIMUM TOTAL LATERAL HEADLOSS IN SYSTEM | 3.42 | | | |
| MANIFOLD HEADLOSS (center-fed unless manifold design) | 0.50 | | | |
| DELIVERY PIPE HEADLOSS | 4.17 | w/ delivery | | 3 inch diameter |
| FITTING LOSS (headloss *.15) | 0.45 | extra if fittings not minimal | | |
| DISTAL PRESSURE HEAD | 3.00 | | | |
| STATIC HEAD (OFF-SWITCH TO HIGH LATERAL/MANIFOLD) | 5.00 | | | |
| HEADLOSS PUMP TO WEEPHOLE (assume 3' run) | 0.13 | | | |
| PUMP MUST BE ABLE TO PASS SOLIDS AT | 139.05 | G.P.M | 16.66 | FEET HEAD |

ORIFICE AND LATERAL SEGMENT DETAILED CALCULATIONS

FRICTION CALCULATIONS (using Hazen Williams friction ft= Ld((3.55Qm/Ch(Dd^2.63)))^1.85)
PRESSURE CALCULATIONS (using orifice dischage equation Q=11.79 D^2 hd^.5
NOTE: Orifices and pipe segments are measured from the end (distal) of lateral

| | Lateral 1: | Lateral 2: | Lateral 3: | Lateral 4: |
|---|---|---|---|---|
| DISTAL ORIFICE DISCHARGE, (GPM)* | 1.28 | 1.28 | 1.47 | 1.47 |
| 1st segment FRICTION, in feet** | 0.00 | 0.00 | 0.00 | 0.00 |
| 2nd ORIFICE DISCHARGE | 1.28 | 1.28 | 1.47 | 1.47 |
| total LATERAL FLOW at this orifice | 2.55 | 2.55 | 2.95 | 2.95 |
| 2nd segment FRICTION | 0.00 | 0.00 | 0.00 | 0.00 |
| 3rd ORIFICE DISCHARGE | 1.28 | 1.28 | 1.47 | 1.47 |
| total LATERAL FLOW at this orifice | 3.83 | 3.83 | 4.42 | 4.42 |
| 3rd segment FRICTION | 0.01 | 0.01 | 0.01 | 0.01 |
| 4TH ORIFICE DISCHARGE | 1.28 | 1.28 | 1.48 | 1.48 |
| total LATERAL FLOW at this orifice | 5.11 | 5.11 | 5.90 | 5.90 |
| 4th segment FRICTION | 0.01 | 0.01 | 0.01 | 0.01 |
| 5TH ORIFICE DISCHARGE | 1.28 | 1.28 | 1.48 | 1.48 |
| total LATERAL FLOW at this orifice | 6.39 | 6.39 | 7.37 | 7.37 |
| 5th segment FRICTION | 0.01 | 0.01 | 0.02 | 0.02 |
| 6TH ORIFICE DISCHARGE | 1.28 | 1.28 | 1.48 | 1.48 |
| total LATERAL FLOW at this orifice | 7.66 | 7.66 | 8.85 | 8.85 |
| 6th segment FRICTION | 0.02 | 0.02 | 0.03 | 0.03 |
| 7TH ORIFICE DISCHARGE | 1.28 | 1.28 | 1.48 | 1.48 |
| total LATERAL FLOW at this orifice | 8.95 | 8.95 | 10.33 | 10.33 |
| 7th segment FRICTION | 0.03 | 0.03 | 0.04 | 0.04 |
| 8TH ORIFICE DISCHARGE | 1.28 | 1.28 | 1.48 | 1.48 |
| total LATERAL FLOW at this orifice | 10.23 | 10.23 | 11.81 | 11.81 |
| 8th segment FRICTION | 0.04 | 0.04 | 0.05 | 0.05 |
| 9TH ORIFICE DISCHARGE | 1.28 | 1.28 | 1.48 | 1.48 |
| total LATERAL FLOW at this orifice | 11.51 | 11.51 | 13.30 | 13.30 |
| 9th segment FRICTION | 0.04 | 0.04 | 0.07 | 0.07 |
| 10TH ORIFICE DISCHARGE | 1.29 | 1.29 | 1.49 | 1.49 |
| total LATERAL FLOW at this orifice | 12.80 | 12.80 | 14.78 | 14.78 |
| 10th segment FRICTION | 0.05 | 0.05 | 0.08 | 0.08 |
| 11TH ORIFICE DISCHARGE | 1.29 | 1.29 | 1.49 | 1.49 |
| total LATERAL FLOW at this orifice | 14.08 | 14.08 | 16.27 | 16.27 |
| 11th segment FRICTION | 0.06 | 0.06 | 0.10 | 0.10 |
| 12TH ORIFICE DISCHARGE | 1.29 | 1.29 | 1.49 | 1.49 |
| total LATERAL FLOW at this orifice | 15.37 | 15.37 | 17.76 | 17.76 |
| 12th segment FRICTION | 0.07 | 0.07 | 0.11 | 0.11 |
| 13TH ORIFICE DISCHARGE | 1.29 | 1.29 | 1.49 | 1.49 |
| total LATERAL FLOW at this orifice | 16.67 | 16.67 | 19.26 | 19.26 |
| 13th segment FRICTION | 0.09 | 0.09 | 0.13 | 0.13 |
| 14TH ORIFICE DISCHARGE | 1.29 | 1.29 | 1.50 | 1.50 |
| total LATERAL FLOW at this orifice | 17.96 | 17.96 | 20.75 | 20.75 |
| 14th segment FRICTION | 0.10 | 0.10 | 0.15 | 0.15 |
| 15TH ORIFICE DISCHARGE | 1.30 | 1.30 | 1.50 | 1.50 |
| total LATERAL FLOW at this orifice | 19.26 | 19.26 | 22.25 | 22.25 |
| 15th segment FRICTION | 0.11 | 0.11 | 0.17 | 0.17 |
| 16TH ORIFICE DISCHARGE | 1.30 | 1.30 | 1.50 | 1.50 |
| total LATERAL FLOW at this orifice | 20.56 | 20.56 | 23.76 | 23.76 |
| 16th segment FRICTION | 0.13 | 0.13 | 0.19 | 0.19 |
| 17TH ORIFICE DISCHARGE | 1.30 | 1.30 | 1.51 | 1.51 |
| total LATERAL FLOW at this orifice | 21.86 | 21.86 | 25.27 | 25.27 |
| 17th segment FRICTION | 0.14 | 0.14 | 0.21 | 0.21 |
| 18TH ORIFICE DISCHARGE | 1.31 | 1.31 | 1.51 | 1.51 |
| total LATERAL FLOW at this orifice | 23.17 | 23.17 | 26.78 | 26.78 |
| 18th segment FRICTION | 0.16 | 0.16 | 0.24 | 0.24 |

| | | | | |
|---|---|---|---|---|
| 19TH ORIFICE DISCHARGE | 1.31 | 1.31 | 1.52 | 1.52 |
| total LATERAL FLOW at this orifice | 24.48 | 24.48 | 28.30 | 28.30 |
| 19th segment FRICTION | 0.18 | 0.18 | 0.26 | 0.26 |
| 20TH ORIFICE DISCHARGE | 1.31 | 1.31 | 1.52 | 1.52 |
| total LATERAL FLOW at this orifice | 25.79 | 25.79 | 29.82 | 29.82 |
| 20th segment FRICTION | 0.20 | 0.20 | 0.29 | 0.29 |
| 21TH ORIFICE DISCHARGE | 1.32 | 1.32 | 1.53 | 1.53 |
| total LATERAL FLOW at this orifice | 27.11 | 27.11 | 31.35 | 31.35 |
| 21th segment FRICTION | 0.21 | 0.21 | 0.32 | 0.32 |
| 22TH ORIFICE DISCHARGE | 1.32 | 1.32 | 1.53 | 1.53 |
| total LATERAL FLOW at this orifice | 28.43 | 28.43 | 32.88 | 32.88 |
| 22th segment FRICTION | 0.23 | 0.23 | 0.35 | 0.35 |
| 23TH ORIFICE DISCHARGE | 1.33 | 1.33 | 1.54 | 1.54 |
| total LATERAL FLOW at this orifice | 29.76 | 29.76 | 34.41 | 34.41 |
| 23th segment FRICTION | 0.25 | 0.25 | 0.38 | 0.38 |
| 24TH ORIFICE DISCHARGE | 1.33 | 1.33 | | |
| total LATERAL FLOW at this orifice | 31.09 | 31.09 | | |
| 24th segment FRICTION | 0.28 | 0.28 | | |
| 25TH ORIFICE DISCHARGE | 1.33 | 1.33 | | |
| total LATERAL FLOW at this orifice | 32.42 | 32.42 | | |
| 25th segment FRICTION | 0.30 | 0.30 | | |
| 26TH ORIFICE DISCHARGE | 1.34 | 1.34 | | |
| total LATERAL FLOW at this orifice | 33.76 | 33.76 | | |
| 26th segment FRICTION | 0.32 | 0.32 | | |

## Appendix Bibliography

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